ROPP

Investigation of a Trunnion Bascule-Bridge

Civil Engineering

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INVESTIGATION OF A TRUNNION BASCULE-BRIDGE

BY

FRANKLIN NEWTON ROPP

THESIS

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IN

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

FRANKLIN NEWTON ROPP

ENTITLED INVESTIGATION OF A TRUNNION BASCULE BRIDGE

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in Civil Engineering

F. O. Dufour
Instructor in Charge.

APPROVED: Ira O. Baker

HEAD OF DEPARTMENT OF Civil Engineering

CONTENTS.

I. INTRODUCTION.

The bridge under investigation is a new style of bascule bridge, and the first one of its kind to be erected for railroad purposes. It crosses the Cuyahoga River in Cleveland, Ohio, on the Wheeling and Lake Erie R.R.; and was thrown open to traffic October 13, 1905.

The bridge was invented and designed by J. B. Strauss, of Chicago, and is known as the Strauss trunnion-bascule bridge.

The principal dimensions of the bridge are shown in Plate I, p. 3.

The bridge consists of the following essential parts; first, a tower supporting the trunnions; second, a moving leaf spanning the river channel with a projection of about one panel length to the land side of the main trunnions; third, a counterweight bearing on the end of the projection of the moving leaf and hinged so as always to retain an upright position; and fourth, an operating mechanism, including an operating strut which is hinged at a point one panel length ahead of the main

trunnion, and which runs over a pinion connected to the operating engines.

This new bridge replaced an old swing bridge which had become inefficient on account of the use of heavier motive power.

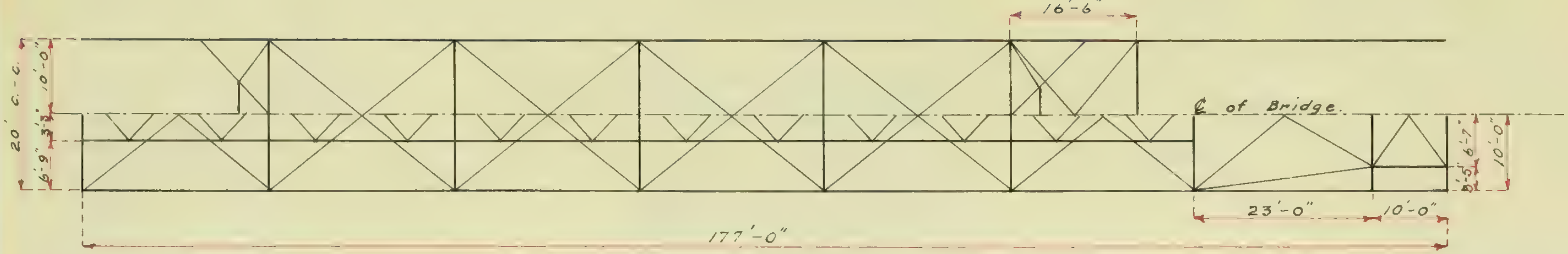
This investigation is made, in accordance with the Wabash Railroad Company's Standard Specifications for Railroad Bridges (1905 Edition), to determine the efficiencies of the different members of the moving leaf under this heavier loading.

PLATE I.

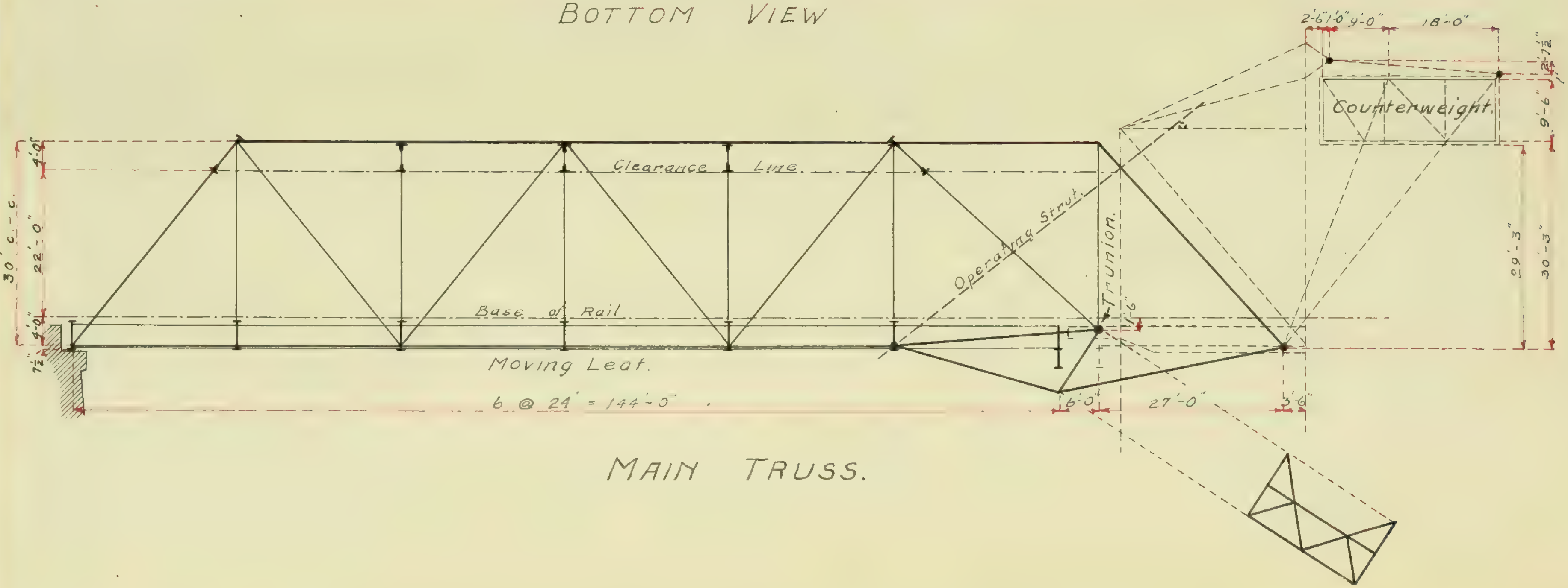
GENERAL NOTES.

Live Load. - 2 Engines - Coopers E 50.
Dead " Wt. of Metal and Floor.
Specifications - Wabash R.R. 1905.
Rivets $\frac{7}{8}$" diam. except as noted.

TOP VIEW.



BOTTOM VIEW



MAIN TRUSS.

GENERAL DIMENSIONS OF BRIDGE.

II. WEIGHT OF METAL AND LUMBER.

The dead load upon the bridge includes the weight of the truss members, the lateral braces, and the floor system. The weight of each member, together with its details, was computed separately and recorded in Table 1, p.p. 6 to 9.

The wood floor consists of 8" by 10" cross ties notched over the flange of the stringers and spaced about 14" center to center. They are 10' and 11'-3" in length and arranged so as to support a two plank footway on each side of the track, as shown in Fig. 1.

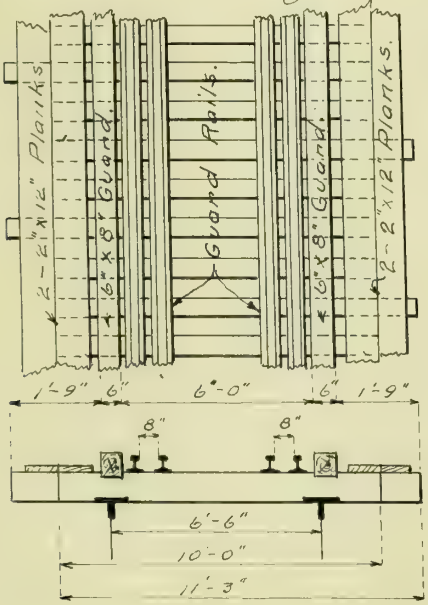


Fig. 1.

Outer guard timbers 6"x8" are notched over the cross ties and are bolted to them and the stringer flange with $\frac{3}{4}$"⌀ x 1'-2" lag bolts.

The total weight of lumber was determined by taking the weight as $4\frac{1}{2}$ pounds per board foot.

Approximately 1300 half pound railroad spikes, 120 bolts at 214.5

pounds per hundred, and 1000 - 30 penny spikes at 4.5 pounds per hundred were used, making a total weight of about 1000 pounds as recorded in Table I, p. 8..

TABLE I. TRUSSES.

Ref. Nº	Name of Piece	Nº. of Pieces.	Weights Main Members.	Details.	Total.
1	$L_0\ U_1$	2	10940	1525	12465
2	$L_0\ L_2$	2	7600	2961	10561
3	$U_1\ L_1$	2	2992	1749	4741
4	$U_1\ U_3$	2	11745	1640	13385
5	$U_1\ L_2$	2	6575	2171	8746
6	$U_2\ L_2$	2	3200	1762	4962
7	$L_2\ U_3$	2	6575	2170	8745
8	$L_2\ L_4$	2	11590	5271	16861
9	$U_3\ L_3$	2	2993	1748	4741
10	$U_3\ U_5$	2	13105	1830	14935
11	$U_3\ L_4$	2	8420	2251	10671
12	$U_4\ L_4'$	2	3200	1762	4962
13	$L_4\ U_5$	2	8420	2250	10670
14	$L_4\ L_5$	2	5530	4195	9725
15	$U_5\ L_5$	2	3700	1625	5325
16	$U_5\ U_6$	2	11155	1372	12527
17	$U_5\ L_6$	2	11600	2676	14276
18	$L_5\ L_6$	2	13048	1980	15028
19	$L_6\ L_8$	2	6130	2426	8556
20	$L_6\ U_6$	2	4320	2505	6825
21	$L_6\ L_8$	2	13576	3239	16815
22	$U_6\ L_7$	2	10556	2539	11095
23	$L_7\ L_8$	2	17291	3722	21013
24	$L_8\ s.$	2			755
				TOTAL –	248385

TAIL END STRUTS.

Ref. Nº	Name of Piece	Nº. of Pieces.	Main Members.	Details.	Total.
25	L_{s_1}	1			
26	L_{s_2}	2	5973	2298	8271.
27	L_{s_3}	2			
				TOTAL –	8271

TABLE I CONT. FLOOR SYSTEM.

Ref. No.	Name of Piece	No of Pieces	Weights. Main Members.	Details.	Total.
	F_1	1	3533	502	4035
	F_2	4	13020	5392	18412
	F_3	1	3255	1348	4603
	F_4	1	2630	375	3005
	End Girder	1	2910	1770	4680
	S_1	1	3861	567	4428
	S_2	1	3861	568	4429
	S_3	1	3861	568	4429
	S_4	1	3861	568	4429
	S_5	3	11583	1705	13288
	S_6	4	15444	2270	17714
	S_7	1	3861	565	4426
	SL_1	23			1868
	SL_2	1			82
	Rails	200 yds			17000
				TOTAL –	86828

TOP LATERAL SYSTEM.

Ref. No.	Name of Piece	No of Pieces	Main Members.	Details.	Total.
	T_1	4			2911
	T_2	4			1507
	T_3	4			2068
	T_4	1			608
	T_5	1			309
	T_6	1			309
	Portal O.E.	1			3147
	" I E	1			3300
	Ts_1	2			3258
	Ts_2	1			1629
	Ts_3	1			1650
				TOTAL –	20696

TABLE I CONT. BOTTOM LATERAL SYSTEM.

Ref. No.	Name of Piece	No. of Pieces	Weights: Main Members	Weights: Details	Weights: Total
	L_1	1			600
	L_2	1			175
	L_3	1			163
	L_4	2			1075
	L_5	2			268
	L_6	2			265
	L_7	2			1002
	L_8	2			232
	L_9	2			230
	L_{10}	1			531
	L_{11}	1			178
	L_{12}	1			178
	L_{13}	1			439
	L_{14}	1			170
	L_{15}	1			114
	L_{16}	1			214
	L_{17}	1			104
	L_{18}	1			102
	L_{19}	2			509
	Sway Strut	1.			3226
				TOTAL	9775

WOOD FLOORING.

Ref. No.	Name of Piece	No. of Pieces	Weights: Main Members	Weights: Details	Weights: Total
	Ties	154			42492
	Guards	16			6000
	Planks				14018
	Bolts & Spikes				300
	R.R. spikes	1300			700
				TOTAL	63510

TABLE I CONT. COUNTERWEIGHT BOX.

Ref. No	Name of Piece	No of Pieces	Weights. Main Members	Details.	Total.
	CB_1	2	8525	2688	11223
	CB_2	2	8390	925	9315
	CB_3	2	6800	2351	9151
	CB_4	2	6230	1705	7935
	CB_5	2	1020	627	1647
	CB_6	2	1760	588	2348
	CB_7	2	765	405	1170
	CB_8	2	350	365	715
	CB_9	2	1340	403	1743
	CB_{10}	2	890	398	1288
	CB_{11}	2	535	391	926
	Portal Struts	$CB_{12}-CB_{17}$			8921
	CB_{18}	1			2765
	CB_{19}	1			2500
	CB_{24}	1.			2100
	CB_{25}	1.			2000
	Floor System	$CB_{26}-CB_{33}$			3265
	Link Strut.	CL_1-CL_7.			16526
				TOTAL -	85538

PINS.

	7" diam	4.			1160
	12" "	2			2620.
				TOTAL -	3780

III DETERMINATION OF STRESSES.

Art. 1. Determination of the Dead Panel Loads.

Owing to the fact that the construction of the members near the trunion becomes heavier, the panel loads are unequal. Plate II is a graphic presentation of the determination of the panel loads. On the lower horizontal line, spaces are laid off to correspond to the panel lengths of the bridge. On the first vertical line is laid off to scale, the weight of all members which go to make the first panel load. The succeeding ordinates represent the weight of the panel load at those points plus all preceeding panel loads.

As no part of the first panel load goes to the upper chord, it is subtracted from the succeeding ordinates. This is accomplished by drawing the horizontal red line through the upper point of the first ordinate.

The red curve is drawn through points on the ordinates located two thirds up from the horizontal red line. The exception is at L_6 where 14.6 kips go entirely to the lower chord, and one third of the remainder only goes to the upper chord. The 14.6 kips must then be subtracted

from the ordinate before the point for the curve is located.

The ordinates above the red curve and between the red curve and horizontal red line show the summation of the loads on the upper and lower chords respectively. The differences between these ordinates and the ones preceding give the loads at that panel point.

The loads as determined, are shown on the diagram Plate III, p. 13.

PLATE II.

DETERMINATION OF PANEL LOAD.

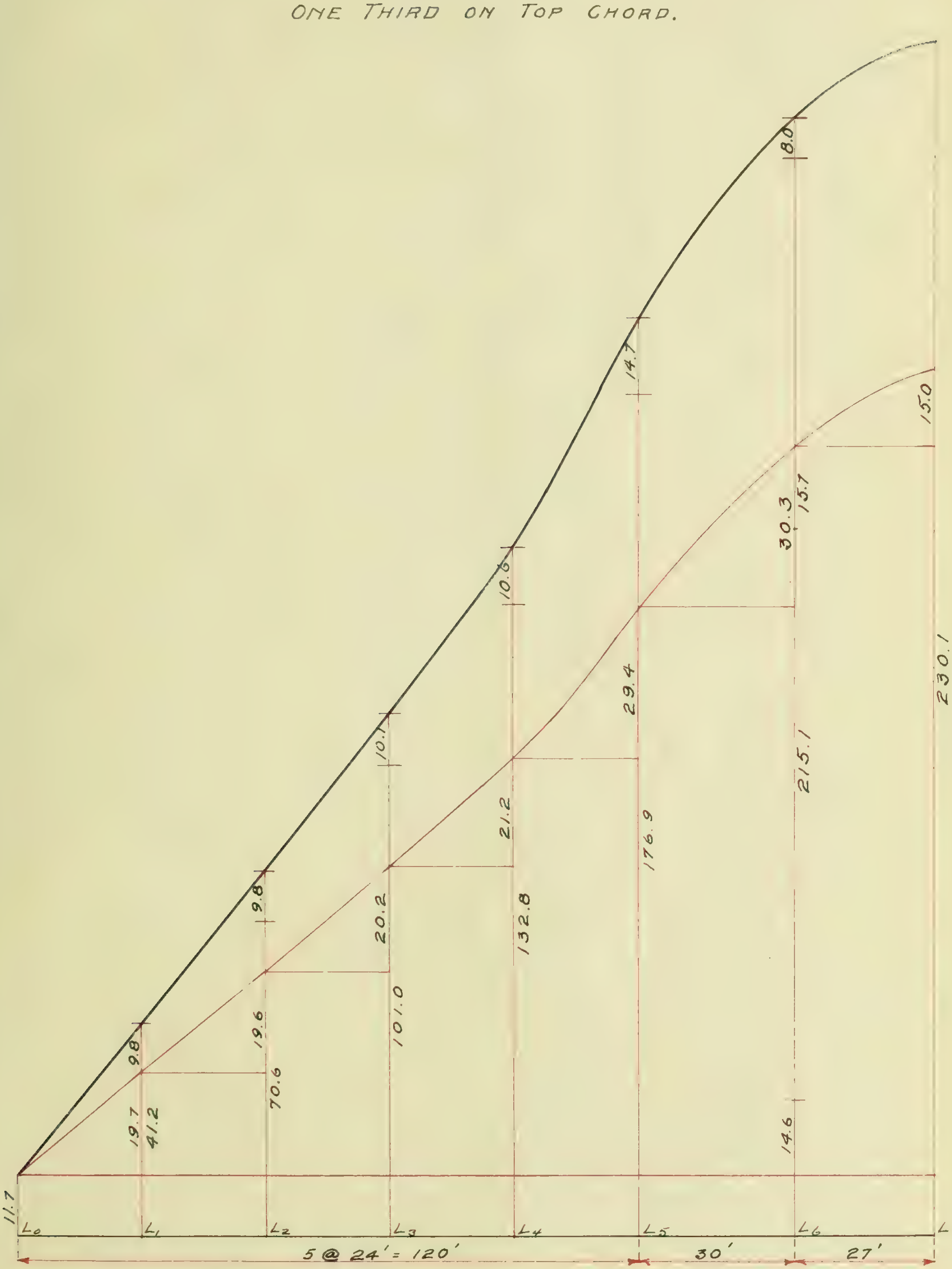
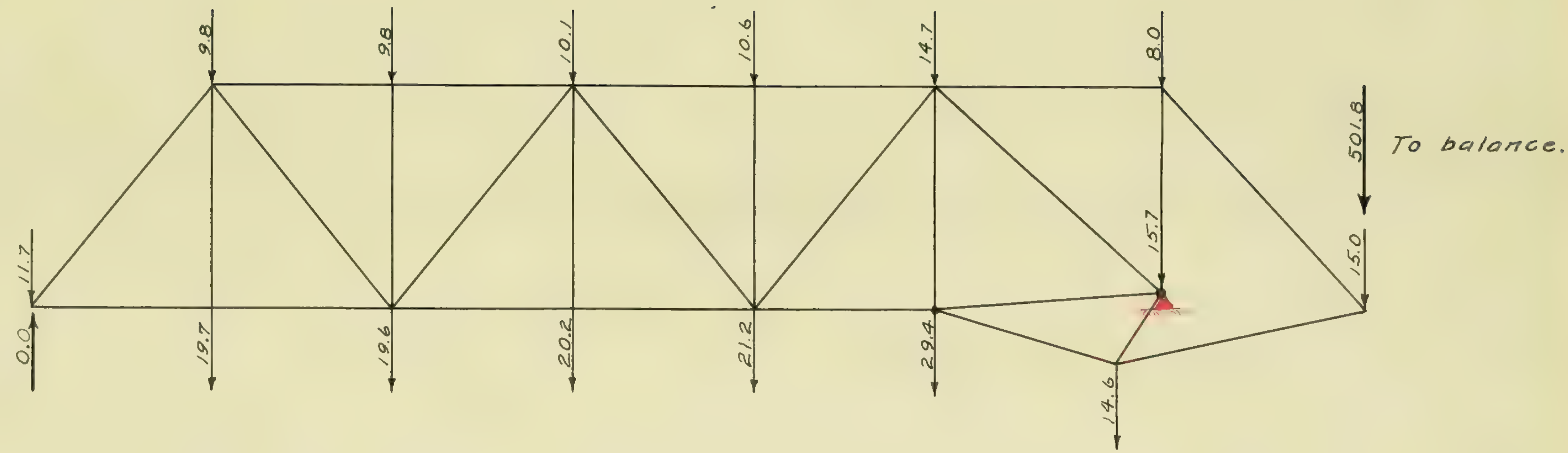


PLATE III.


9.8
9.8
10.1
10.6
14.7
8.0
501.8
To balance.
15.7
15.0
11.7
0.0
19.7
19.6
20.2
21.2
29.4
14.6

Art. 2. Dead Load Stresses, Bridge Closed.

To determine the dead load stresses in the members, the amount and effect of the counterweight had to be determined first. The moments of all known loads were taken about the trunion as a center, to determine the vertical force applied at the end of the projecting panel necessary to cause equilibrium. This force known, the amount of the counterweight, the force exerted by it on the bridge, and the direction of this force, as well as the stresses in the counterweight posts and link, were determined graphically.

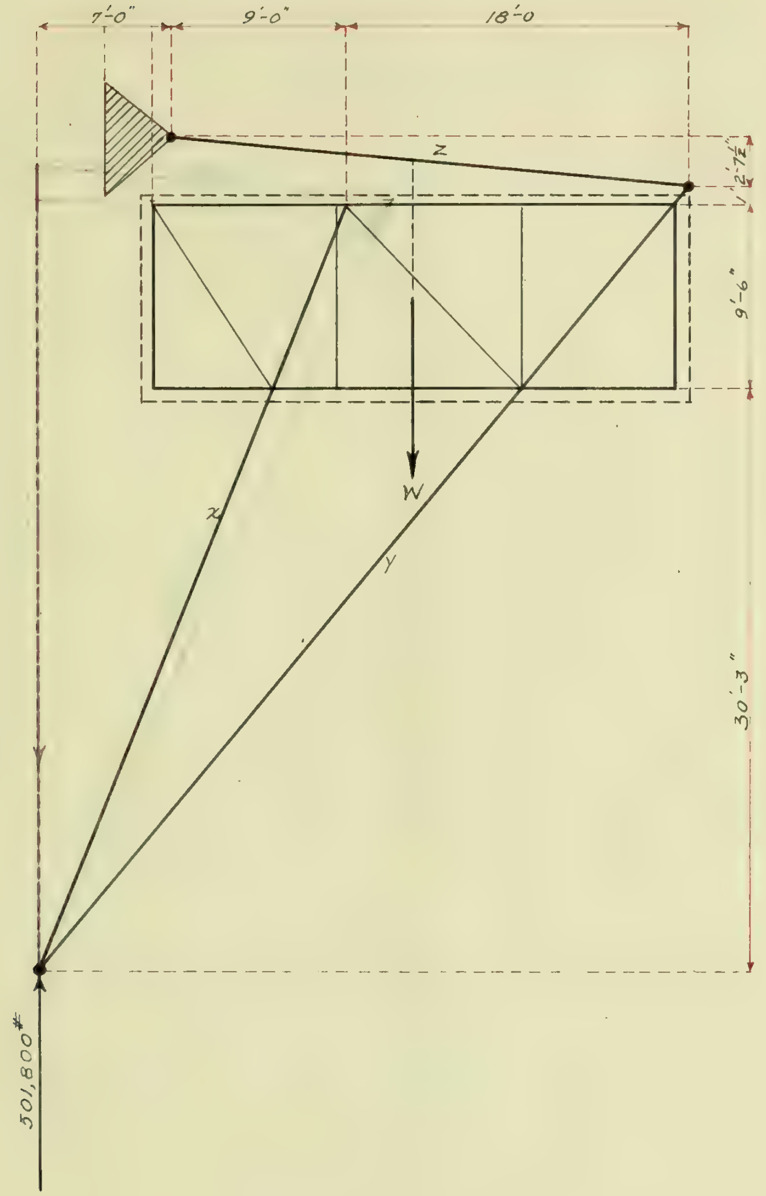
The stresses in the members of the first five panels were determined by algebraic methods, but for the stresses in the remaining members, the graphic method was used.

The stresses, as determined, are shown on the truss in Plate V, p. 17.

PLATE IV.

COUNTERWEIGHT POSTS.

Bridge closed.



STRESSES.

x = 465000

y = 89000

Z = 230500

W = 522000

TABLE II.

STRESSES.

$$Sec. \theta = \frac{\sqrt{30^2+24^2}}{30} = 1.28$$

DIAGONALS.

SHEARS.	MEMBERS.	STRESSES.
11.7 Kips.	$L_0\ U_1$	− 14960
41.2 "	$U_1\ L_2$	+ 52700
70.6 "	$L_2\ U_3$	− 90250
100.9 "	$U_3\ L_4$	+128700
132.7 "	$L_4\ U_5$	−169800

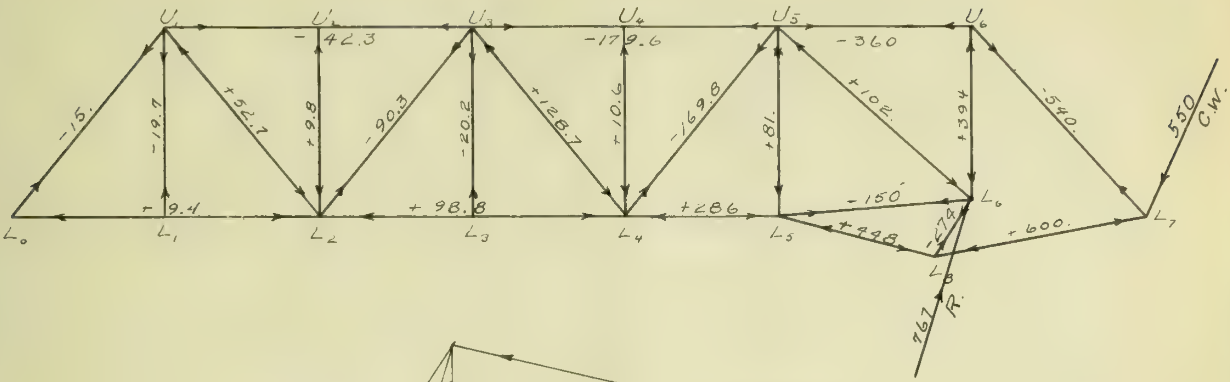
CHORDS.

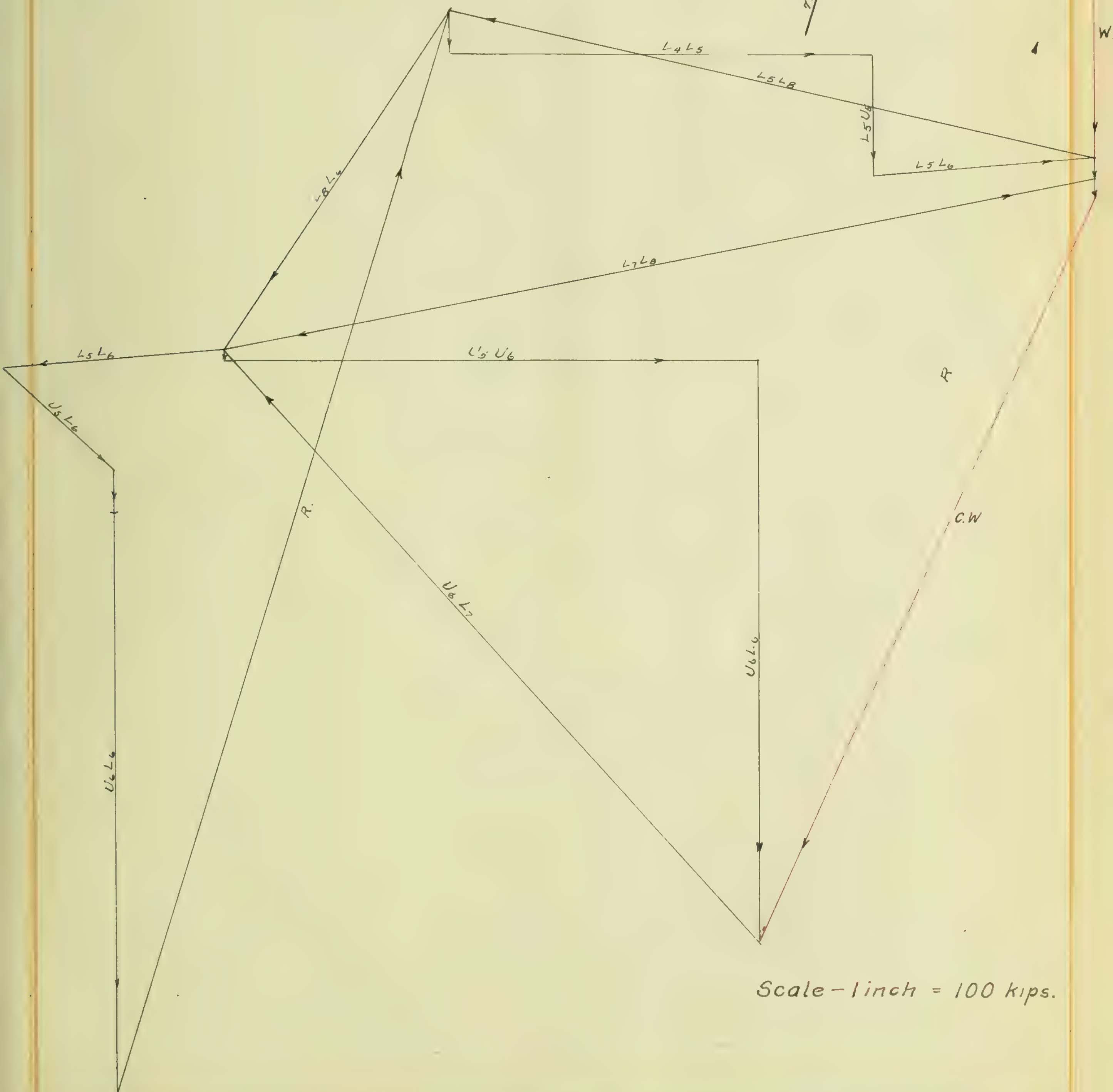
C.m. U_1	$L_0\ L_2$	+ 9350
" " L_2	$U_1\ U_3$	− 42300
" " U_3	$L_2\ L_4$	+ 98800
" " L_4	$U_3\ U_5$	−179600

Other stresses determined graphically.

PLATE V.

DEAD LOAD STRESSES.





Scale–1 inch = 100 kips.

Art. 3. Dead Load Stresses, Bridge Open 90°.

Using the value of the counterweight as obtained in art. 2, the direction and value of the force exerted by it on the bridge was determined graphically.

The stresses in the members in panels 7, 6, and 5 were determined graphically, while those for the chord members above were found by taking the weights which come upon them; the web members being considered as taking no stress.

The results, as determined, are shown in Plates VI and VII, p.p. 19 and 20.

PLATE VI.

COUNTERWEIGHT POSTS.

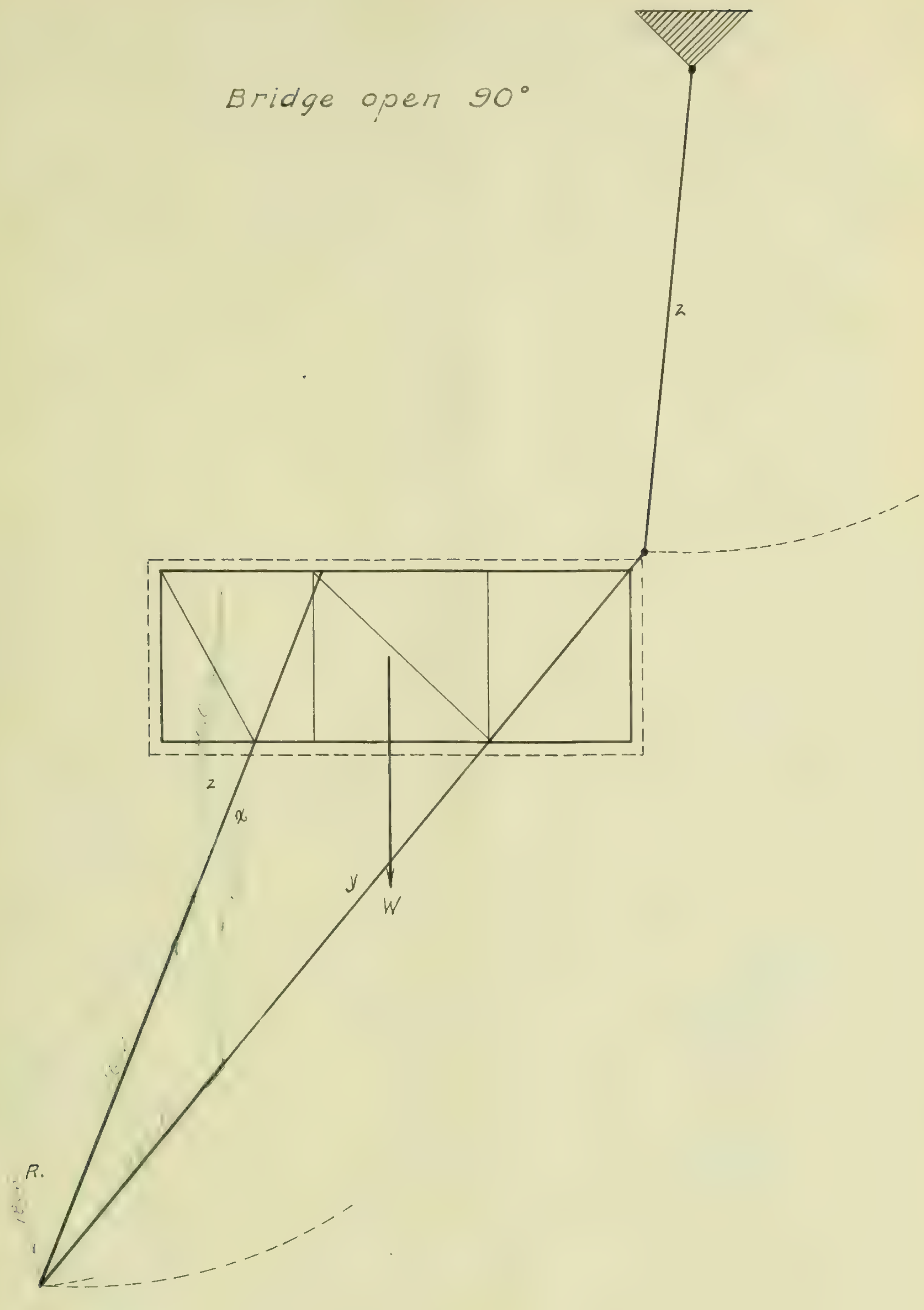
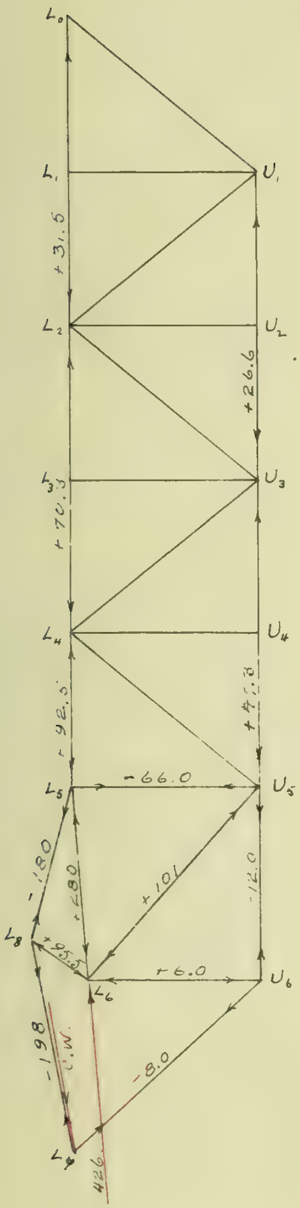
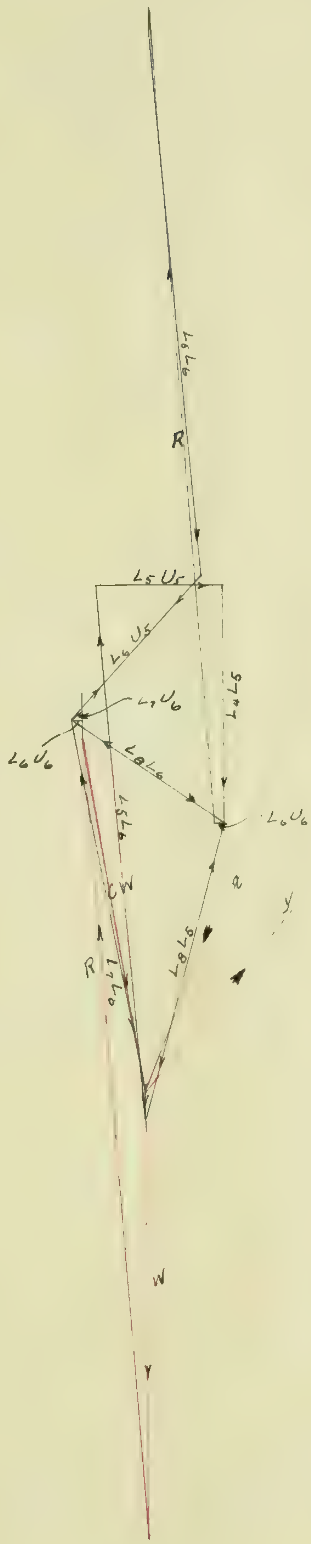


PLATE VII.

DEAD LOAD STRESSES.

Bridge opened 90°.





Scale – 1 inch = 100 kips.

Art. 4. Live Load Stresses.

The maximum shears and moments in the different panels were determined using an engine diagram, E 40 loading, and these were multiplied by $\frac{5}{4}$ to give the respective maximum shears and moments under E.50 loading, as shown in Tables III, IV and V, p.p.

Owing to a difference in panel lengths, it was necessary to load the bridge from each side in order to determine the maximum moments and positive and negative shears.

The stresses were in most cases determined algebraically, but in some cases graphic methods were used.

The calculations and stresses are shown in Table VI, p. 25.

TABLE III.

MAXIMUM LIVE LOAD SHEAR. RIGHT.

Point	Wheel	G	W/m.	G+P.		E 40	E 50
L_1	2	10	$334 \cdot \frac{24}{150} = 53.5$	30			
"	3	30	$344 \cdot$ " $= 55.0$	50	$R \left\{ \begin{array}{r} 25784 \\ 25 \\ \underline{1720} \\ \frac{27529}{150} \end{array} \right. = 183.5$		
"	4	50	$354 \cdot$ " $= 56.5$	70			
"	5	70	$384 \cdot$ " $= 61.5$	90	$r = -\frac{480}{24} = -\underline{20.}$		
					$V_1 =$	+ 163.5	+ 204.0
L_2	2	10	$286 \cdot \frac{24}{150} = 45.7$	30			
"	3	30	$296 \cdot$ " $= 47.3$	50	$R \left\{ \begin{array}{r} 16364 \\ 36 \\ \underline{1704} \\ \frac{18104}{150} \end{array} \right. = 120.7$		
"	4	50	$306 \cdot$ " $= 48.9$	70			
					$r = -\frac{230}{24} = \underline{-9.6}$		
					$V_2 =$	+ 111.1	+ 138.9
L_3	2	10	$232 \cdot \frac{24}{150} = 37.0$	30			
"	3	30	$245 \cdot$ " $= 39.2$	50	$R \left\{ \begin{array}{r} 10816 \\ \underline{735} \\ \frac{11551}{150} \end{array} \right. = 77.0$		
"	4	50	$258 \cdot$ " $= 41.4$	70			
					$r = -\frac{230}{24} = -\underline{9.6}$		
					$V_3 =$	+ 67.4	+ 84.2
L_4	1	0	$142 \cdot \frac{24}{150} = 22.7$	10			
"	2	10	$152 \cdot$ " $= 24.3$	30	$R \left\{ \begin{array}{r} 4632 \\ \underline{912} \\ \frac{5544}{150} \end{array} \right. = 37.0$		
"	3	30	$172 \cdot$ " $= 27.5$	50			
					$r = -\frac{80}{24} = -\underline{3.33}$		
					$V_4 =$	+ 33.6	+ 42.0
L_5	1	0	$90 \cdot \frac{24}{150} = 14.4$	10			
"	2	10	$116 \cdot$ " $= 18.5$	30	$R \left\{ \begin{array}{r} 2155 \\ \underline{116} \\ \frac{2271}{150} \end{array} \right. = 15.1$		
"	3	30	$116 \cdot$ " $= 18.5$	50			
					$r = -\frac{80}{24} = \underline{-3.3}$		
					$V_5 =$	+ 11.8	+ 14.7

TABLE IV.

MAXIMUM LIVE LOAD SHEAR. LEFT.

Point	Wheel	G	W/m.		G+P		E 40	E 50.
L_4	2	10	$271 \cdot \frac{24}{150} =$	43.3	30			
"	3	30	$284 \cdot \frac{24}{150} =$	45.5	50	$R = \frac{16364}{150} = 109.1$		
"	4	50	$294 \cdot \frac{24}{150} =$	48.0	70	$r = -\frac{230}{24} = -9.6$		
						$V =$	99.5	124.5
L_3	2	10	$232 \cdot \frac{24}{150} =$	37.0	30			
"	3	30	232 · " =	37.0	50	$R = \begin{cases} 8728 \\ 1392 \\ \frac{10120}{150} = 67.5 \end{cases}$		
"	4	50	245 · " =	39.2	70			
						$r = -\frac{230}{24} = -9.6$		
						$V =$	57.9	72.4
L_2	1	0	$129 \cdot \frac{24}{150} =$	20.7	10			
"	2	10	142 · " =	22.7	30	$R. = \frac{4632}{150} = 30.9$		
"	3	30	152 · " =	24.3	50	$r = -\frac{80}{24} = -3.3$		
						$V =$	27.6	34.5
L_1	1	0	$90 \cdot \frac{24}{150} =$	14.4	10			
"	2	10	90 · " =	14.4	30	$R = \frac{1640}{150} = 10.9$		
"	3	30	103 · " =	16.5	50	$r = -\frac{80}{24} = -3.3$		
						$V =$	7.6	9.5

TABLE V. MAXIMUM LIVE LOAD MOMENTS.

Point	Wheel	G/l	W/m	$\frac{G+P}{l}$	Moments.	E 40	E 50.
L_1	4	50	$342 \cdot \frac{24}{150} = 54.5$	70	25784 25 1720 $27529 \cdot \frac{24}{150} - 480 =$	3925.	4910
L_2	5	35	$316 \cdot \frac{24}{150} = 50.5$	45			
"	6	45	334 · " = 53.5	51.5			
"	7	51.5	344 · " = 55.0	58	$25784 \cdot \frac{48}{150} - 2155 =$	6096	7620.
"	8	58.	356 · " = 57.0	64.5			
L_3	9	43.	$318 \cdot \frac{24}{150} = 51.0$	47.33	25784 9 1032 $26825 \cdot \frac{72}{150} - 5848 =$	7026	8775
"	10	47.33	334 · " = 53.5	50.67			
"	11	50.67	350 · " = 56.0	57.33			
"	12	57.33	360 · " = 57.6	64.00	25784 64 2752 $28600 \cdot \frac{72}{150} - 6708 =$	6992	
"	13	64.00	370 · " = 59.2	70.67			
L_4	12	43	$312 \cdot \frac{24}{150} = 47.8$	48			
"	13	48	322 · " = 50.7	53	19304 81 2736 $22121 \cdot \frac{96}{150} - 7668 =$	6482	8130
"	14	53	332 · " = 53.0	58			
"	15	58	350 · " = 56.0	61.25	22444 16 1296 $23756 \cdot \frac{96}{150} - 8728 =$	6472	
			LOADED FROM THE LEFT.				
L_5	4	50	$342 \cdot \frac{30}{150} = 68.4$	70	22444 81 2916 $25441 \cdot \frac{30}{150} - 480$	4608.2	5760
	5	70	$352 \cdot \frac{30}{150} = 70.4$	90	25784 16 1376 $27176 \cdot \frac{30}{150} - 830$	4605.2	

TABLE VI. LIVE LOAD STRESSES.

MEM.	WHEEL	AT	LOADED FROM.	CALCULATION	MEM.	STRESS
L_0U_1	4	L_1	Right.	$204 \times 1.28 - L_0U_1 = 0$	1	+261000
L_0L_2	4	L_1	"	$4910 + 30\ L_0L_1 = 0$	2	−163700
U_1L_1	4	L_1	"	Max Floor Beam Reaction	3	− 92500
U_1U_3	7	L_2	"	$7620 - 30\ U_1U_3 = 0$	4	+254000
U_1L_2	3	L_2	"	$138.9 \times 1.28 + U_1L_2 = 0$	5	−177500
"	2	L_1	Left.	$-9.5 \times 1.28 + U_1L_2 = 0$	5	+ 12200
L_2U_3	3	L_3	Right.	$84.2 \times 1.28 - L_2U_3 = 0$	7	+108000
"	2	L_2	Left	$-34.5 \times 1.28 - L_2U_3 = 0$	7	− 44200
L_2L_4	11	L_3	Right	$8775 + 30\ L_2L_4 = 0$	8	−292500
U_3L_3	4	L_3	"	Max. Floor Beam Reaction	9	− 92500
U_3U_5	13	L_4	"	$8130 - 30\ U_3U_5 = 0$	10	+271000
U_3L_4	2	L_4	"	$42.0 \times 1.28 + U_3L_4 = 0$	11	− 54000
"	3	L_3	Left.	$-72.4 \times 1.28 + U_3L_4 = 0$	11	+ 92500
L_4U_5	2	L_5	Right.	$14.7 \times 1.28 - L_4U_5 = 0$	13	+ 19000
"	3	L_4	Left.	$-124.5 \times 1.28 - L_4U_5 = 0$	13	−160000
L_4L_5	4	L_5	"	$5760 + 30\ L_4L_5 = 0$	14	−192000
U_5L_5	4	L_5	Right.	Graphic −	15	− 92000
"	2	L_4	Left.	Graphic. − $L_5L_6 \sin\theta - U_5L_5 = 0$	15	+ 7300
U_5L_6	4	L_5	"	"	17	+260000
L_5L_6	4	L_5	"	"	18	−193000
L_5L_8			"	" End V. 27′-10″ Stringer	19	− 45000
L_8L_6			"	" " " "	21	− 78000

Art. 5. Impact Stresses.

The impact stresses were computed by the formula.-

$$I = \frac{300}{L+300} S,$$

where I is the impact stress,
S is the maximum live load stress, and L is the loaded of bridge when maximum S occurs.

The results are given in Table VII, p. 27.

TABLE VII. IMPACT STRESSES.

MEM.	WHEEL	AT	LOADED FROM	L.	$\frac{300}{L+300}$	L.L. STRESS	MEM.	IMPACT
L_0U_1	4	L_1	Right	144'	.675	+261000	1	+176000
L_0L_2	4	L_1	"	"	"	-163700	2	-110000
U_1L_1	4	L_1	"	42	.870	- 92500	3	- 81000
U_1U_3	7	L_2	"	139	.683	+254000	4	+174000
U_1L_2	3	L_2	"	115	.723	-177500	5	-128000
"	2	L_1	Left.	32	.905	+ 12200	5	+ 11000
L_2U_3	3	L_3	Right	91	.777	+108000	7	+ 83000
"	2	L_2	Left.	56	.842	- 44200	7	- 37300
L_2L_4	11	L_3	Right	,42	.670	-292500	8	-198000
U_3L_3	4	L_3	"	42	.870	- 92500	9	- 81000
U_3U_5	13	L_4	"	128	.700	+271000	10	+190000
U_3L_4	2	L_4	"	62	.828	- 54000	11	- 44600
"	3	L_3	Left	85	.780	+ 92500	11	+ 72000
L_4U_5	2	L_5	Right.	38	.889	+ 19000	13	+ 16900
"	3	L_4	Left.	109	.735	-160000	13	-117500
L_4L_5	4	L_5	"	138	.674	-192000	14	-131400
U_5L_5	4	L_5	Right	45	.070	- 92000	15	- 80000
"	2	L_4	Left	104	..743	+ 7300	15	+ 5500
U_5L_6	4	L_5	"	138	.683	+260000	17	+178000
L_5L_6	4	L_5	"	,38	.683	-193000	18	-132000
L_5L_8	2	L_5	"	36	.895	- 45000	19	- 40500
L_8L_6	2	L_5	"	36	895	- 78000	21	- 70000

Art. 6. Machinery Stresses.

The so called machinery stresses are merely the stresses due to the wind when the bridge is held open 90° by the operating strut. The stress may be either tension or compression, depending on the direction of the wind.

The full wind panel load was assumed as four thousand pounds.

Owing to the simplicity of the graphic solution, it was used in preference to the more complicated algebraic solution in determining the stresses in the members about the trunnion.

The calculations and results are shown in Table VIII, p. 29.

TABLE VIII. MACHINERY STRESSES.

WIND PANEL LOAD = 4000 Lbs.

MEMBER	SHEAR	MOMENT	CALCULATION.	MEM.	STRESS.
$L_0 U_1$	2000		$2000 \times 1.28 \pm L_0U_1 = 0$	1	± 2600
$L_0 L_2$		48000	$48000 \div 30 \pm L_0L_2 = 0$	2	± 1600
$U_1 L_1$	4000		$4000 \times 1.0 \pm U_1L_1 = 0$	3	± 4000
$U_1 U_3$		192000	$192000 \div 30 \pm U_1U_3 = 0$	4	± 6400
$U_1 L_2$	6000		$6000 \times 1.28 \pm U_1L_2 = 0$	5	± 7700
$L_2 U_3$	10000		$10000 \times 1.28 \pm L_2U_3 = 0$	7	± 12800
$L_2 L_4$		432000	$432000 \div 30 \pm L_2L_4 = 0$	8	± 14400
$U_3 L_3$	4000		$4000 \times 1.0 \pm U_3L_3 = 0$	9	± 4000
$U_3 U_5$		768000	$768000 \div 30 \pm U_3U_5 = 0$	10	± 25600
$U_3 L_4$	14000		$14000 \times 1.28 \pm U_3L_4 = 0$	11	± 18000
$L_4 U_5$	18000		$18000 \times 1.28 \pm L_4U_5 = 0$	13	± 23000
$L_4 L_5$		1200000	$1200000 \div 30 \pm L_4L_5 = 0$	14	± 40000
$U_5 L_5$			Graphic – Forces about U_5	15	± 54000
$U_5 L_6$			Graphic – " " L_6	17	± 54000
$L_5 L_6$			Graphic – " " L_6	18	± 40000
$L_5 L_8$			Graphic – " " L_8	19	± 1200
$L_8 L_6$			Graphic – " " L_8	21	± 2000
OP. STRUT.		1350000	$1350000 \div 30 \pm OP.S. = 0$	24	± 45000

Art. 7. Maximum and Allowable Unit Stresses.

Plate VIII, p. 31, shows the reference numbers of the members as used in Tables IX and X, pp. 32 and 33.

In combining the stresses to obtain the maximum, it must be noted that the machinery and dead load stresses, occurring when the bridge is open, cannot be used unless their sum is greater than the sum of the live load, impact, and other dead load stresses.

When the dead load stress is opposite in sign to the live load and impact stresses, only one half of the dead load stress is used in combining.

The maximum stresses occurring in the members are given in Table IX, p. 32.

Table X shows the composition of the members, their gross area, length in inches, least radius of gyration, and allowable unit stress in pounds per square inch for tension and compression.

PLATE VIII.

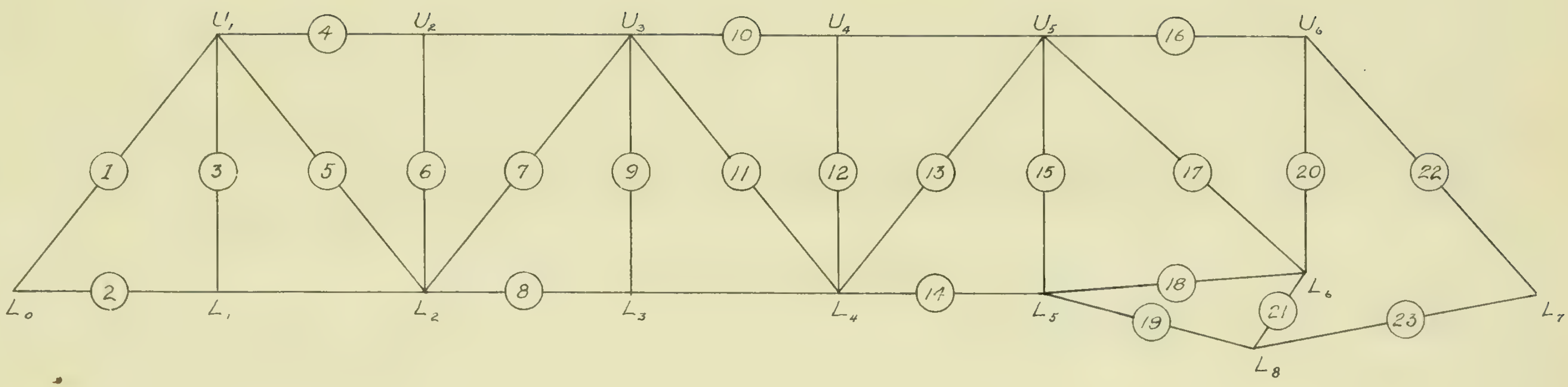

U1
U2
U3
U4
U5
U6
L0
L1
L2
L3
L4
L5
L6
L7
L8
1
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TABLE IX.

TABLE OF STRESSES.

MEMBER	DEAD LOAD.		LIVE LOAD		IMPACT.		MACHINERY.	MAXIMUM.	
	DOWN.	UP-90°	+	–	+	–	±	+	–
1	- 15000		261000		176000		+ 2600	427500	15000
2	+ 9400	+ 31500		163700		110000	1600	33100	269000
3	- 19700			92500		81000	4000	4000	193200
4	- 42300	+ 26600	254000		174000		6400	406900	42300
5	+ 52700		12200	177500	11000	128000	7700	75900	279200
6	+ 9800							9000	
7	- 90300		108000	44200	83000	37300	12800	145800	171800
8	+ 98800	+ 70300		292500		198000	14400	98000	441100
9	- 20200			92500		81000	4000	4000	193700
10.	-179600	+ 47300	271000		190000		25600	371200	179600
11	+128700		92500	54000	72000	44600	18000	293200	57100
12	+ 10600	- 12000						10600	12000
13	-169800		19000	160000	16900	117500	23000	23000	447300
14	+286000	+ 92500		192000		131400	40000	286000	280400
15	+ 81000	- 66000	7300	92000	5500	80000	54000	93800	131500
16	-360000								360000
17	+102000	+101000	260000		178000		54000	540000	
18	-150000	+280000		193000		132000	40000	320000	475000
19	+448000	-180000		45000		40500	1200	387200	181200
20	+394000	+ 6000						394000	
21	-274000	+ 95500		78000		70000	2000	95500	422000
22	-540000	- 8000							540000
23	+600000	-198000						600000	198000
24.							45000	45000	45000

TABLE X. ALLOWABLE UNIT STRESSES.

MEMBER.	SECTION.	AREA SQ. IN.	L IN.	R. IN.	ALL. UNIT STRESS +	ALL. UNIT STRESS −
1.	2 [s 15" @ 50# 1 Pl. 22" x $\frac{5}{8}$"	43.18	461	5.65	10300	16000
2.	2 [s 15" @ 40#	23.52	288	5.58	12390	"
3.	2 [s 12" @ 25#	14.70	360	4.43	10320	"
4.	2 [s 15" @ 40# 1 Pl. 22" x $\frac{9}{16}$"	35.90	288	5.75	12490	"
5.	2 [s 15" @ 45#	26.48	461	5.32	9930	"
6.	2 [s 12" @ 25#	14.70	360	4.43	10320	"
7.	2 [s 15" @ 45#	26.48	461	5.32	9930	"
8.	2 [s 15" @ 45# 2 Pl. 12" x $\frac{3}{8}$"	35.48	288	4.90	11880	"
9.	2 [s 12" @ 25#	14.70	360	4.43	10320	"
10.	2 [s 15" @ 45# 1 Pl. 22" x $\frac{5}{8}$"	40.24	288	5.65	12420	"
11.	2 [s 15" @ 55#	32.36	460	5.16	9770	"
12	2 [s 12" @ 25#	14.70	360	4.43	10320	"
13.	2 [s 15" @ 55#	32.36	460	5.23	9850	"
14	2 [s 15" @ 45# 2 Pl. 12" x $\frac{3}{8}$"	35.48	288	4.90	11880	"
15	2 [s 15" @ 33#	19.80	360	5.62	11520	"
16	2 [s 15" @ 33# 4 ∠s $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{1}{2}$"					
	1 Pl. 22" x $\frac{3}{8}$" 1 Pl. $13\frac{1}{2}$" x $\frac{3}{8}$"	46.12				"
17	2 Pl. 21" x $\frac{3}{4}$" 4 ∠s $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{1}{2}$"	48.00	480	7.30	11400	"
18	2 Pl. 21" x $\frac{3}{4}$" 4 ∠s $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{3}{4}$"	53.76	3.60	7.30	12550	"
19	2 Pl. 21" x $\frac{9}{16}$" 4 ∠s $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{9}{16}$"	40.36	300	7.30	13130	"
20	2 Pl. 21" x $\frac{3}{8}$" 4 ∠s $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{3}{8}$"	27.18	324	7.70	13050	"
21	2 Pl. 19" x $\frac{3}{8}$" 1 Pl. 13" x $\frac{3}{8}$"					
	8 ∠s $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{3}{8}$"	38.90	132	7.70	14800	"
22	2 [s 15" @ 45# 1 Pl. 22" x $\frac{7}{16}$"	36.10				
23	2 Pl. 19" x $\frac{5}{8}$" 2 Pl. 12" x $\frac{11}{16}$"					
	4 ∠s $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{5}{8}$"	56.18	408	6.80	11800	"

IV. SYNOPSIS OF SPECIFICATIONS.

In General.

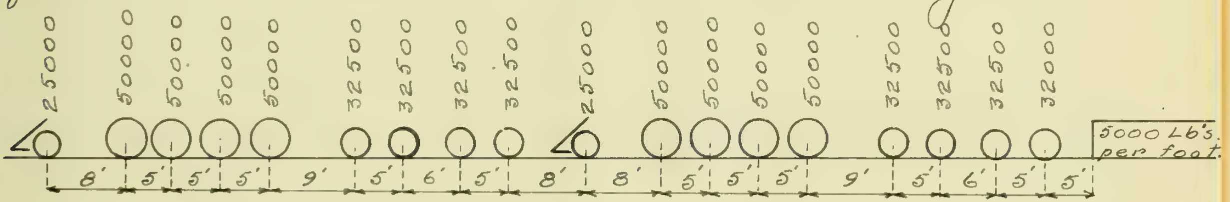
1. The plus (+) sign is to be used for "compression".

2. The minus (−) sign is to be used for "tension"

Loading.

3. The dead load is to be taken as the weight of metal and lumber in the bridge.

4. The live load is to be taken as two coupled consolidated E 50 engines and tenders, followed by a uniform train load of 5000 pounds per foot, as shown in the diagram.



5. Impact is to be figured from the formula; $I = \frac{300}{L+300} S$, where

I = the impact stress,

S = maximum live load stress, and

L = loaded length of bridge when the maximum live load stress occurs.

6. The wind loads are to be taken as 600 and 300 pounds per linear foot of bridge for the lower and upper chords respectively, and are to be

treated as moving loads.

Unit Stresses.

7. Axial tension (net section)..................................16000 Lbs. per square inch.

8. Compression (gross section)

$$P = 16000 - 70\frac{l}{r},$$

where "l" = unsupported length, in inches, and "r" = least radius of gyration, in inches.

9. Bending (extreme fiber)..................................16000 Lbs. per square inch.

10. Shearing.—
Shop rivets......12000 Lbs. per square inch
Field " ....9000 " " " "
Pins.................12000 " " " "
Plate girder webs........ $12000 - 40\frac{d}{t}$,
where "d" is the minimum unsupported distance in inches, between the flange angles or stiffeners, and "t" is the thickness of the web.

11. Bearing.—
Shop rivets......24000 Lbs per square inch.
Field " ....18000 " " " "
Pins.................24000 " " " "

12. Members subject to alternate tension and compression must be designed to resist either stress increased by $\frac{8}{10}$ of the smaller stress.

V. Investigation of Efficiencies.

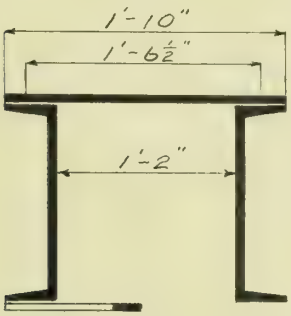
Almost every member of the bridge is subject to a reversal of stress, and this necessitates the use of the increased stress in the investigation, as stated in § 12 in the synopsis of specifications, p. 34.

The detailed investigation of the members is given in the following articles.

Art. 8. Truss Members.

The stresses are taken from Table IX, p. 32, and the unit stresses from Table X, p. 33.

(1.) Member L_0U_1:- The area required for compression is $\frac{427,500 + .8 \times 15000}{10300} = 42.67$ square inches (gross section). The efficiency of the member is then found by dividing the actual gross area, 43.18 square inches, by the required gross area which gives $\frac{43.18}{42.67} = 1.035.$



2 [s 15" @ 50#
1 Pl. 22" x 5/8"

Fig. 2.

The actual net area of the member is 36.63 square inches, while for tension, only $\frac{15000 + .8 \times 15000}{16000}$ or 1.62 square inches required, giving an efficiency of $\frac{36.63}{1.62} = 22.60.$

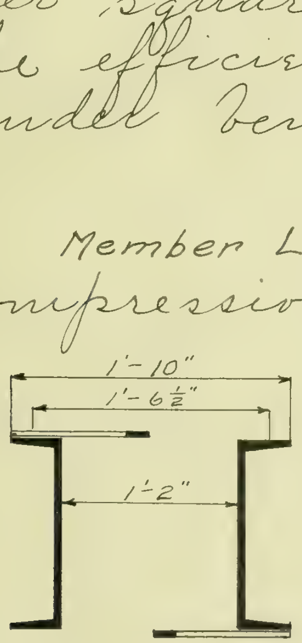
The rivets in the end connection at L_0 develop a strength of $70 \times 7220 = 505{,}400$ pounds, giving an efficiency of $\frac{505{,}400}{427{,}500} = 1.18$.

At U_1, the rivets develop a strength of $84 \times 5415 = 455{,}400$ pounds which gives an efficiency of $\frac{455{,}400}{427{,}500} = 1.062$.

The bending moment at the foot of the knee brace is about 2,300,000 inch pounds, and from the formula, $S = \frac{Mc}{I - \frac{Pl^2}{10E}}$, the maximum fiber stress is found to be 9450 pounds per square inch. The allowable fiber stress is 16000 pounds per square inch, which makes the efficiency of this member under bending $\frac{16000}{9450} = 1.70$.

(2) Member L_0L_2:— The area required for compression is $\frac{33{,}100 + .8 \times 33{,}100}{12390} = 4.82$ square inches. The actual area of the member is 23.52 square inches, and the efficiency for compression is, therefor, $\frac{23.52}{4.82} = 4.82$.

The net area of the section is 19.33 square inches, and as $\frac{269{,}000 + .8 \times 33{,}100}{16000} = 18.4$ square inches



2 [15" @ 40#

Fig. 3.

are required, the efficiency of the member in tension is $\frac{19.33}{18.40} = 1.05$

The strength developed by the rivets at $L_0 = 70 \times 5415 = 379,050$ pounds, and as the maximum stress in the member is only 269,000 pounds, the efficiency is $\frac{379,050}{269,000} = 1.41$.

At L_2, the rivets develop a strength of $36 \times 5415 + 24 \times 8253 = 393,012$ pounds, giving an efficiency of $\frac{393,012}{269,000} = 1.46$.

In a similar manner, the efficiencies of the other truss members, together with their connections, are determined. They are given in Table XI, p. 39.

TABLE XI. EFFICIENCIES.

MEMBER	COMPRESSION	TENSION	END CONNECTIONS.			
			END		END.	
$L_0 U_1$	1.03	22.60	L_0	1.18	U_1	1.06
$L_0 L_2$	4.82	1.05	L_0	1.41	L_2	1.46
$U_1 L_1$	21.00	1.01	U_1	1.12	L_1	
$U_1 U_3$	1.02	6.65	U_1	1.34	U_3	1.21
$U_1 L_2$	1.92	1.01	U_1	1.43	L_2	1.43
$U_2 L_2$	9.38		U_2	9.63	L_2	2.35
$L_2 U_3$	1.00	1.19	L_2	2.21	U_3	2.27
$L_2 L_4$	2.39	.85	L_2	1.52	L_4	1.34
$U_3 L_3$	21.00	1.00	U_3	1.08	L_3	
$U_3 U_5$	.97	1.75	U_3	1.69	U_5	1.46
$U_3 L_4$	.93	4.02	U_3	1.15	L_4	1.18
$U_4 L_4$	7.92		U_4	7.21	L_4	2.35
$L_4 U_5$	7.70	.92	L_4	1.09	U_5	1.09
$L_4 L_5$	.83	.87	L_4	2.17	L_5	1.46
$U_5 L_5$	1.35	1.14	U_5	1.98	L_5	3.75
$U_5 U_6$		1.15	U_5	1.40	U_6	.97
$U_5 L_6$	1.01		U_5	.98	L_6	1.04
$L_5 L_6$	1.17	.97	L_5	4.85	L_6	1.11
$L_5 L_8$	.99	1.64	L_5	1.64	L_8	1.36
$U_6 L_6$	.90		U_6	.99	L_6	.99
$L_6 L_8$	3.35	1.10	L_6	1.37	L_8	1.13
$U_6 L_7$		.92	U_6	.95	L_7	.86
$L_7 L_8$	.88	2.18	L_7	1.82	L_8	1.65

Art. 9. Floor Beams.

In this bridge there are five intermediate and two end floor beams of similar construction.

As the intermediate floor beams support the loads in two panels, the stresses in them will be much larger than in the end floor beams, and if they prove efficient, the others will also, therefore, only the intermediate ones will be investigated.

The floor beam consists of a $54\frac{1}{2}'' \times \frac{3}{8}''$ web plate and $6'' \times 6'' \times \frac{13}{16}''$ flange angles, as shown in Figure 4.

Fig. 4.

At the lower corner of the floor beam a portion is cut away in order to clear the lower chord, and to secure enough for rivets in the end connecting angles, a part of the web must be extended beyond the upper flange. This is accomplished by splicing a smaller web plate to the main web plate, as shown in Figure 5.

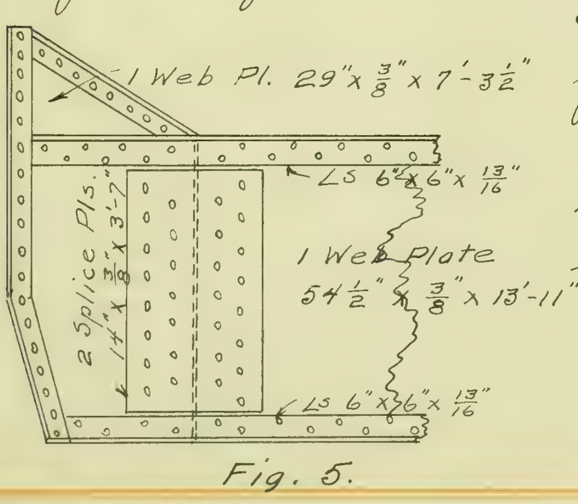


Fig. 5.

The maximum end shear is equal to the sum of the shears due to live and dead load and impact, which are 92,500, 9109. and 81,000 pounds respectively, giving 182,609 pounds. The strength of the web in shear is $54\frac{1}{2} \times \frac{3}{8} \times (12000 - 40 \frac{44.5}{.375}) = 148,000$ pounds which gives an efficiency of $\frac{148,000}{182,609} = .81$.

The rivets in the splice develop a strength of $27 \times 7875 = 212,625$ pounds, giving an efficiency of $\frac{212,625}{182,609} = 1.16$.

In the connection of the angles to the web plate, the 26 rivets give an efficiency of 1.12.

The 40 rivets in single shear, which connect the floor beam to the post, give an efficiency of $\frac{40 \times 7,220}{182,609} = 1.58$.

The maximum bending moment in the floor beams occurs at the stringer connection and equals approximately $74 \times (92500 + 81000 + 4429) + \frac{1}{8} \times 4680 \times 226 =$ 13,298,200 inch pounds. The net area of the flange required to withstand this stress, is equal to the maximum moment divided by the product of the allowable unit stress and the moment

arm of the flange, in inches, which equals $\frac{13,298,200}{51.4 \times 16000} = 16.2$ square inches. The required net area of the angles equals the net flange area minus the web equivalent $= 16.2 - \frac{54.5 \times 3}{8 \times 8} =$ 13.64 square inches. The actual net area of the flange angles is 17.12 square inches which makes the efficiency $= \frac{17.12}{13.64} = 1.25$.

Art. 10. Stringers.

The stringers in this bridge are all very nearly alike, the only difference being in the details. They consist of $42'' \times \frac{3}{8}''$ web plates and $6'' \times 6'' \times \frac{5}{8}''$ flange angles, with $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle stiffeners spaced about 3 feet apart. Two $6'' \times 6'' \times \frac{1}{2}''$ angles are used at the end connections.

The maximum shear in the stringer at the end and is equal to the sum of the live and dead load shears and impact, which is 69250 + 2215 + 62750, or 134,215 pounds. The stringer web will stand $42 \times \frac{3}{8} \times (12000 - 40\frac{30}{.375})$ $= 138400$ pounds which gives for it an efficiency of $\frac{138,400}{134,215} = 1.03$.

The rivets in the connecting

angles and stringer web develop a strength of $17 \times 7875 = 133,875$ pounds giving an efficiency of $\frac{133,875}{134,215} = .995$.

In the connection between the stringer and the floor beam, the number of rivets required to develop a strength equal to the end shear of the floor beam due to live load and impact, plus the end shear of the stringer due to dead load, is $\frac{175,715}{7875} = 23$, which is the maximum. There are actually 34, which makes the efficiency $\frac{34}{23} = 1.48$.

From the tables in specifications, the maximum live load moment in the stringer is found to be 4,278,000 inch pounds, the impact moment equals .905 the live load moment, or 3,875,000 inch pounds, and the dead load moment $= \frac{1}{8} \times 4430 \times 288$, or 160,000 inch pounds, making 8,313,000 inch pounds for the maximum moment.

The net flange area required $= \frac{8,313,000}{38.54 \times 16000} = 13.5$ square inches. Subtracting the web equivalent or 1.97 leaves 11.53 square inches as the net area required for the flange angles.

The actual net area equals 12.97 square inches, therefore, the efficiency is $\frac{12.97}{11.53} = 1.13$.

Art. 11. Trunion Bearings.

The pressure on the trunion is equal to the sum of the pressures due to dead load, live load, and impact, or 1,140,250 pounds.

The bearing area required is $\frac{1,140,250}{24000} = 43.34$ square inches. The area furnished by the tower support $= 2 \times 20 \times \left(\frac{9}{16} + \frac{9}{16} + \frac{3}{8}\right) = 60$ square inches, which gives an efficiency of $\frac{60.0}{43.34} = 1.38$.

The area furnished by the gusset plates of the truss equal $2 \times 20 \times \left(\frac{3}{4} + \frac{3}{4} + \frac{5}{8}\right) = 85$ square inches, and this gives an efficiency of $\frac{85.0}{43.34} = 1.96$.

VI. CONCLUSION.

Although some of the efficiencies of the members and their connections are less than 100 percent, they are not low enough to be considered serious; the factors of safety used in determining the allowable unit stresses being large enough to easily take care of these reduced efficiencies.

The smallest efficiency occurs in the lower chord member of the fifth panel and, theoretically, if failure occurs it will be in this member.

In taking panel loads, small differences in the assumed loads cause differences in the value of the counterweight which are proportional to the distances of the assumed loads from the trunion. A small increase in the assumed dead panel loads over the actual at the free end of the bridge, cause a multiplied increase in the required value of the counterweight, and as the stresses in the members around the trunion are caused directly by the counterweight, they

vary as the counterweight and necessarily as the panel loads.

The smaller efficiencies are found in the members around the trunion, which is probably due to the fact that larger panel loads at the free end of the bridge, were used by the investigator than by the designer.

The efficiencies are all about 100 percent which indicates that it is at least of economical design, and under ordinary loading it is safe to say that it is safe.